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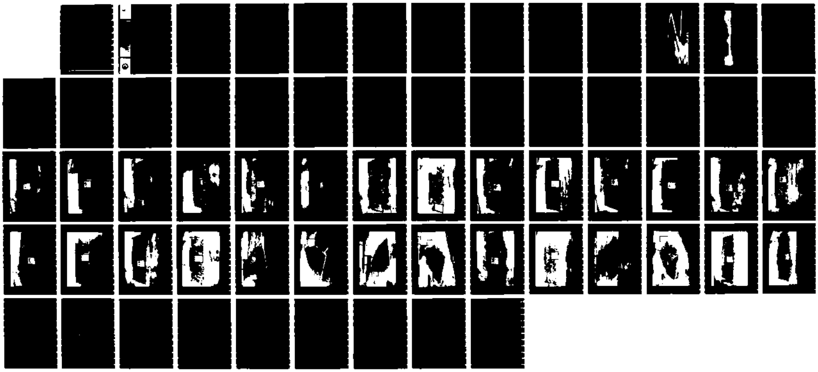

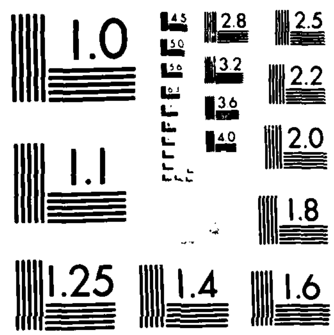

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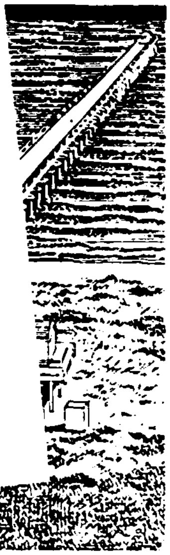

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MISCELLANEOUS PAPER CERC-86-8

STABILITY TEST OF MODIFIED REPAIR OPTIONS FOR SAN PEDRO BREAKWATER LOS ANGELES, CALIFORNIA

Coastal Model Investigation

by

R. Clay Baumgartner, Robert D. Carver,
D. Donald Davidson, Charles R. Herrington

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

July 1986

Final Report

Approved For Public Release, Distribution Unlimited

Prepared for

US Army Engineer District, Los Angeles
PO Box 2711
Los Angeles, California 90053

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704 0188
Exp Date Jun 30 1986

AD-A171 876

1a REPORT SECURITY CLASSIFICATION: Unclassified

1b RESTRICTIVE MARKINGS

2a SECURITY CLASSIFICATION AUTHORITY

2b DECLASSIFICATION / DOWNGRADING SCHEDULE

3 DISTRIBUTION / AVAILABILITY OF REPORT: Approved for public release: distribution unlimited.

4 PERFORMING ORGANIZATION REPORT NUMBER(S): Miscellaneous Paper CERC-86-8

5 MONITORING ORGANIZATION REPORT NUMBER(S)

6a NAME OF PERFORMING ORGANIZATION: See reverse side

6b OFFICE SYMBOL (If applicable): WESCW

7a NAME OF MONITORING ORGANIZATION

6c ADDRESS (City, State, and ZIP Code): PO Box 631, Vicksburg, MS 39180-0631

7b ADDRESS (City, State, and ZIP Code)

8a NAME OF FUNDING SPONSORING ORGANIZATION: See reverse side

8b OFFICE SYMBOL (If applicable)

9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER

8c ADDRESS (City, State, and ZIP Code): PO Box 2711, Los Angeles, CA 90053

10 SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO	PROJECT NO	TASK NO	WORK UNIT ACCESSION NO

11 TITLE (Include Security Classification): Stability Test of Modified Repair Options for San Pedro Breakwater, Los Angeles, California; Coastal Model Investigation

12 PERSONAL AUTHOR(S): R. Clay Baumgartner, Robert D. Carver, D. Donald Davidson, Charles R. Herrington

13a TYPE OF REPORT	13b TIME COVERED	14 DATE OF REPORT (Year, Month, Day)	15 PAGE COUNT
Final report	FROM Apr 1985 TO Aug 1985	July 1986	63

16 SUPPLEMENTARY NOTATION: Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17 COSATI CODES

FIELD	GROUP	SUB GROUP

18 SUBJECT TERMS (Continue on reverse if necessary and identify by block number)

Breakwater--Model (LC)
San Pedro Breakwater (Los Angeles, CA) (LC)

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

An undistorted scale hydraulic model study was conducted to investigate the stability response of modified repair options for the San Pedro Breakwater. A plan which employs [illegible]- to [illegible]-ton armor stone was developed. This plan proved to be more stable than the original breakwater section for both [illegible]- and [illegible]-deg wave attack.

20 DISTRIBUTION / AVAILABILITY OF ABSTRACT: ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS

21 ABSTRACT SECURITY CLASSIFICATION: Unclassified

22a NAME OF RESPONSIBLE INDIVIDUAL

22b TELEPHONE (Include Area Code)

22c OFFICE SYMBOL

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

6a. NAME OF PERFORMING ORGANIZATION (Continued).

USAEWES
Coastal Engineering Research Center

8a. NAME OF FUNDING/SPONSORING ORGANIZATION (Continued).

US Army Engineer District, Los Angeles

PREFACE

The model investigation described herein was requested by the US Army Engineer District, Los Angeles (SPL), in a meeting at the US Army Engineer Waterways Experiment Station (WES) on 8 January 1985. Funding authorization from SPL was granted in SPL Intra-Army Order No. E86 85-0043, dated 29 January 1985.

Model tests were conducted at WES during the period from April to August 1985 under the general direction of Dr. J. R. Houston, Chief, Coastal Engineering Research Center (CERC), Mr. C. C. Calhoun, Jr., Assistant Chief, CERC, Mr. C. E. Chatham, Chief, Wave Dynamics Division, and Mr. D. D. Davidson, Chief, Wave Research Branch. Tests were conducted by Messrs. R. D. Carver and R. C. Baumgartner, Research Hydraulic Engineers, and Mr. C. R. Herrington, Engineering Technician, and Mrs. L. W. O'Neal, Engineering Aide; Mr. Herrington served as Lead Technician under the immediate supervision of Messrs. Carver and Baumgartner. This report was prepared by Messrs. Baumgartner, Carver, Davidson, and Herrington.

During the study Messrs. Paul Berger and Tad Nazinski and Mrs. Laurie Ruh-Hanson of SPL and Mr. Hugh Converse of US Army Engineer Division, South Pacific, visited WES to observe model operation and provide input relative to the course of testing.

During publication of this report, COL Dwayne G. Lee, CE, was Commander and Director. Dr. Robert W. Whalin was Technical Director.

A-1

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (US statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2,000 lb, mass)	907.1847	kilograms

STABILITY TEST OF MODIFIED REPAIR OPTIONS
FOR SAN PEDRO BREAKWATER
LOS ANGELES, CALIFORNIA

Coastal Model Investigation

PART I: INTRODUCTION

The Prototype

1. The San Pedro Breakwater, located in San Pedro Bay on the southern coast of California (Figure 1), is one of three breakwaters providing protection against wave attack for the Ports of Los Angeles and Long Beach. Typical breakwater cross sections are shown in Figure 2. During the winter of 1982-1983 the San Pedro Breakwater was subjected to severe wave attack in concert with exceptionally high still-water levels (swl's) and incurred some extensive damage (Figures 3 and 4). Immediate repair of the structure was found necessary to prevent further deterioration and to reestablish acceptable wave protection for the harbor. Model tests were conducted at the US Army Engineer Waterways Experiment Station (WES) from June to September 1983 to determine an adequate repair plan.* Based on results of these tests, a molded concrete-block repair section and a stone rubble-mound repair section that are more stable than the laid-up granite block section of the existing breakwater were developed. The stone rubble-mound repair option was chosen for use in the prototype. However, due to temporal constraints, the repair section constructed in September 1983 was considered to be temporary and differed significantly from the section developed in the original model study. The repair section performed satisfactorily during the winter seasons of 1983-1984 and 1984-1985. Wave conditions during these periods were less than the projected design conditions, thus it was questioned whether the temporary repair section could serve as permanent wave protection.

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* Robert D. Carver. 1984. "San Pedro Breakwater Repair Study, Los Angeles, California," Miscellaneous Paper CERC-84-11, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

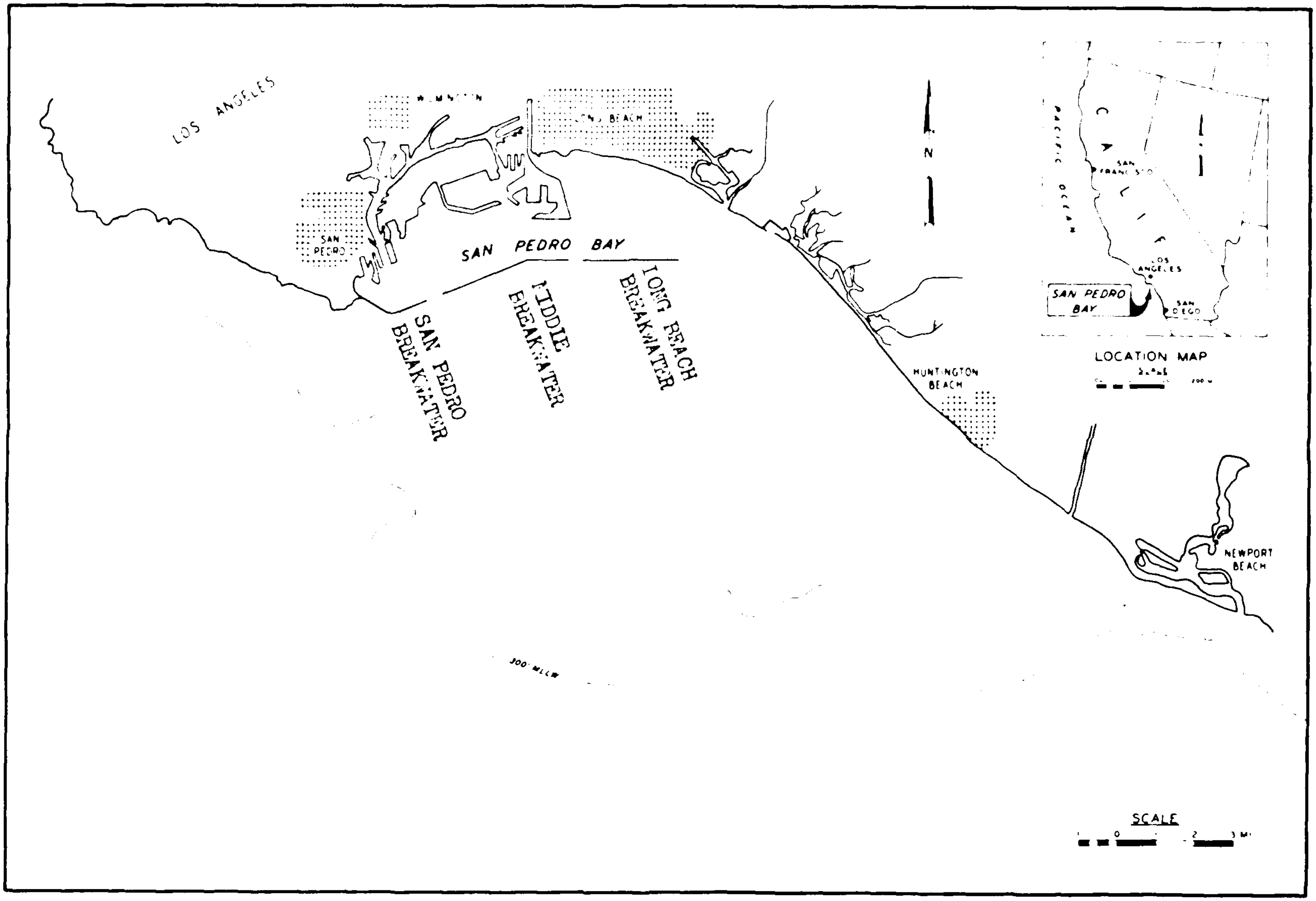

Figure 1. Site map

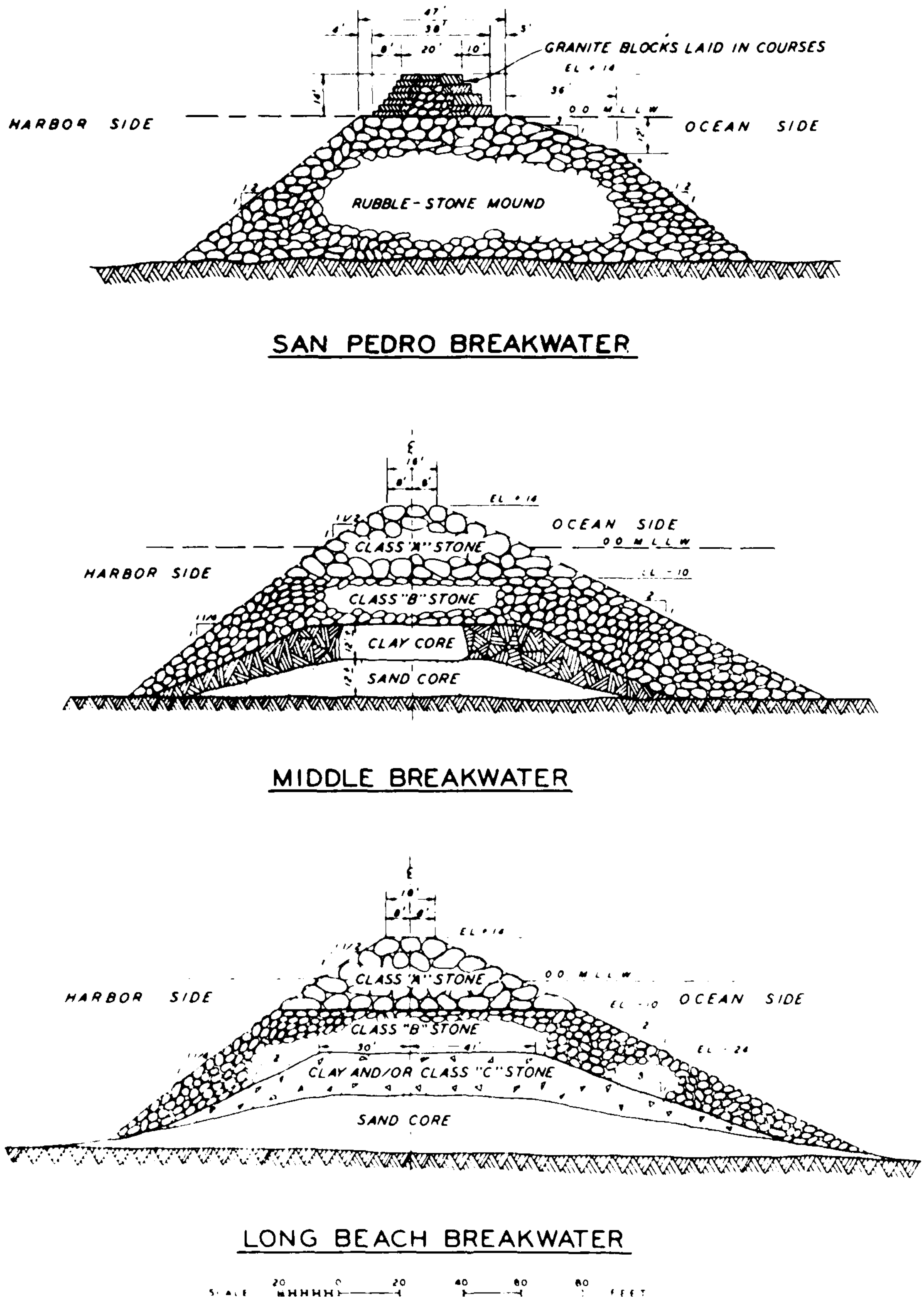

Figure 2. Typical cross sections of the Los Angeles-Long Beach Harbors breakwater

Figure 3. A general view of San Pedro Breakwater after the 1982-1983 winter storm season

Figure 4. Closeup view of damage incurred during the winter of 1982-1983

Purpose and Approach of Model Study

2. The purpose of the model study was to investigate the stability of the rubble-mound repair section as constructed in the prototype. If the repair section proved less stable than the original laid-up granite block construction, modifications or alternate plans would be developed.

PART II: THE MODEL

Design of Model

3. Tests were conducted at a geometrically undistorted linear scale of 1:30, model to prototype. Scale selection was determined by the (a) absolute size of model breakwater sections necessary to ensure the preclusion of stability scale effects,* (b) capabilities of the available wave generator, (c) size of model material available, and (d) depth of water at the toe. Based on Froude's model law** and the linear scale of 1:30, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model-Prototype Scale Relation
Length	L	$L_r = 1:30$
Area	L^2	$A_r = L_r^2 = 1:900$
Volume	L^3	$V_r = L_r^3 = 1:27,000$
Time	T	$T_r = L_r^{1/2} = 1:5.48$

4. The specific weight of water used in the model was assumed to be 62.4 pcf;† that of seawater is 64.0 pcf. Speci c weights of model breakwater construction materials were not identical to their prototype counterparts. These variables were related using the following transference equation:

$$\frac{\left(W_a\right)_m}{\left(W_a\right)_p} = \frac{\left(\gamma_a\right)_m}{\left(\gamma_a\right)_p}\left(\frac{L_m}{L_p}\right)^3 \left[\frac{\left(S_a\right)_p - 1}{\left(S_a\right)_m - 1}\right]^3$$

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* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York.

† A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

where

W_a = weight of an individual armor unit or stone, lb

$_m$ and $_p$ = model and prototype quantities, respectively

γ_a = specific weight of an individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed; i.e., $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

Test Facilities and Equipment

5. Tests were conducted in an L-shaped concrete wave flume (Figure 5) that is 250 ft. long, 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep. Test sections were located on the flat bottom portion of the flume approximately 120 ft from a paddle-type wave generator capable of producing sinusoidal waves of various periods and heights. Changes in water surface elevation as a function of time were measured by electrical wave-height gages and recorded on chart paper by an electrically operated oscillograph. The electrical output of each wave gage was directly proportional to its submergence depth.

Model Construction

6. The prototype bottom slope immediately seaward of the San Pedro Breakwater varies from mild to practically flat. Based on this mild prototype topography and the fact that the average depth (-47 ft mean lower low water (mllw)) at the breakwater is sufficient to prevent depth-limited wave conditions, the model foreslope was flat for all tests conducted. All model breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing prototype breakwaters. Base mound material was dampened with water as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from previous wave attack on the breakwater. Final grade of the base mound was controlled to ±0.5 ft, prototype. Armor stone in the superstructure was individually placed, and the top elevation of each layer was controlled to ±0.5 ft. Final crest elevation of the structure was +13.0 ft mllw (±0.5 ft).

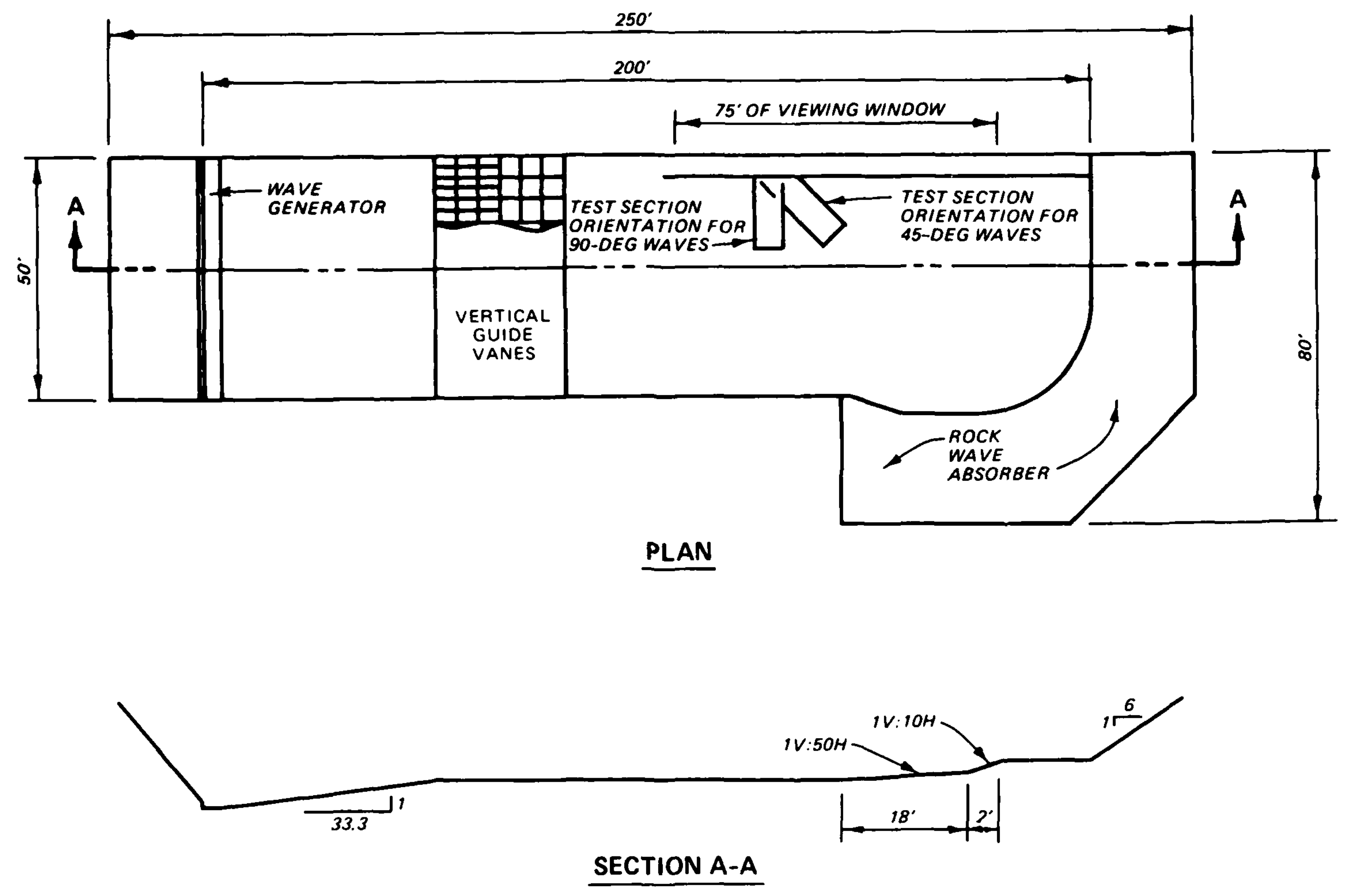

Figure 5. Wave flume layout

After each test, armor units of the superstructure were removed from the breakwater, the base mound material was regraded, and the armor was replaced.

7. Existing armor stone was placed together as closely as possible with vertical joints staggered. Generally, the longitudinal axis of each outer unit was placed normal to the axis of the breakwater and sloped slightly downward toward the center of the breakwater. Core stone within the superstructure was placed progressively with the capstone; i.e., after each layer of capstone had been placed, the core stone for the next capstone was placed.

Method of Reporting Damage

8. The following list of adjectives, in order of increasing severity, was used for recording model observations and reporting test results for each test section: (a) slight, (b) minor, (c) moderate, (d) significant, (e) major, and (f) extensive. Slight and minor were used to describe acceptable results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allows for some quantification of the severity of resulting damage incurred by the breakwater's primary cover-layer units. By using the descriptive adjectives and the before- and after-test photographs, comparisons can be made between alternative test sections.

PART III: TESTS AND RESULTS

Selection of Test Conditions

9. Evaluation of surge and wave data from storms that attacked the San Pedro Breakwater during January, February, and March 1983 revealed a maximum surge of +7.96 ft mllw; consequently, this value was rounded slightly, and a maximum surge of +8.0 ft mllw was selected for simulation in the model. A minimum surge of 0.0 ft mllw was investigated, and an intermediate surge of +6.0 ft mllw also was tested.

10. Measured wave data showed significant energy concentration in the 6- to 22-sec-period range. Wave periods of 6, 8, 11, 16, and 22 sec were selected for testing, thus allowing investigation of a broad range of wave steepness (H/L). Sunset Beach, California, proved to be the closest location for which measured wave data were available for January through March 1983.* The Sunset Beach gage is located about 6 miles south of the San Pedro Breakwater in a depth of 27 ft.

11. The most severe winter storm of the 1982-1983 season occurred 1 March 1983, producing a significant wave height (H_s) equal to 12.9 ft at the Sunset Beach gage. A refraction analysis was performed during the 1983 model study to determine energy distributions and directionality of the 1 March storm at the San Pedro Breakwater. This analysis was accomplished using an iterative scheme that varied deepwater wave direction until approaches at the Sunset Beach gage were within ± 2 deg of the directions determined from the 1 March storm. Corresponding wave directions at the breakwater were approximately 90 deg relative to the shoreward leg of the structure and 45 deg relative to the major breach that occurred on the outer part of the breakwater. Therefore, it was decided to test all plans for 90-deg wave attack, and those plans that showed the best stability response were also tested at a 45-deg angle. Refraction analysis predicted a significant wave height of 15.6 ft. In view of the assumptions inherent in refraction theory, a slightly larger design wave height of 16 ft was selected.

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* US Army Corps of Engineers, "Coastal Data Information Program," monthly reports for January, February, and March 1983, State of California Department of Boating and Waterways, Sacramento.

Description of Plans and Test Results

12. Since the storms of 1982-1983 varied with the swl, wave period, and wave height, Hydrographs A and B, Table 1, were selected for testing. Hydrograph A is an exploratory hydrograph with a minimum number of swls while Hydrograph B is a more generalized representation of the severe storms with additional intermediate swls.

13. Four plans were tested for 90-deg wave attack (wave direction 1), and one of these was also tested at a 45-deg angle (wave direction 2). The model section was a 300-ft-wide prototype, thus allowing the simulation of a 150-ft-wide repair section bound on the sides by 75-ft sections of the existing structure. The plans were built from a toe elevation of -47 ft mllw to a crown elevation of +13 ft mllw. Details of the plans tested and their stability response are described in the following paragraphs.

Tests for a 90-deg angle of wave attack

14. Plan 1 (Photos 1-4 and Plates 1-4) was constructed to model the 1983 repair of the breakwater. The rubble mound repair (Plate 4) consisted of three to four layers of 7- to 20-ton angular stone placed randomly, with one-half of the repair section including the bottom layer of the sea side existing stone and some corestone. The sea-side toe of the rubble-mound section was placed a distance of one angular stone out from the existing granite stone, i.e., approximately 25 ft horizontally from the center line of the breakwater. The harbor-side toe of the repair section was placed 18 ft out from the center line of the breakwater, the same distance out as the toe of the existing stone. At the transitions (Plate 3), existing granite-stone sections were overlaid with one layer of 7- to 20-ton angular stone for a distance of four to five stones. The test section was subjected to Hydrograph B (Table 1). Wave attack at the 0-ft mllw caused minor damage to the rubble-mound section with most of the displacement occurring during attack of the 8-sec, 16-ft waves (Step 5) and originating at the toe of the section. Wave attack at this swl produced no damage to the existing granite-stone sections. Testing at the +6 ft mllw caused severe damage to the rubble-mound section with major damage being initiated at Step 6 (6-sec, 14-ft waves) and continuing for Step 7 (8-sec, 16-ft waves). Damage originated at the transition areas of the rubble-mound section and then spread throughout the

region. Most of the displacement occurred on the sea side of the structure, but some stone shifted on the harbor side as voids were created because of stone lost on the sea side and from wave overtopping. Damage to the existing granite stone was initiated during attack of 8-sec, 16-ft waves and damage to this section was moderate after completion of testing at the +6 ft swl. Movement of the existing granite stone was severe during wave attack at the +8 ft swl. This area of the structure failed completely during testing at this water depth. Displacement of the angular stone continued during testing at the +8 ft swl, but the rate of damage was less than that for the existing granite stone and there was some angular stone remaining after completion of testing at this swl. Testing was stopped at Step 9 (8-sec, 16-ft waves) of Hydrograph B because it was deemed unnecessary to continue after the complete failure of the existing granite block sections. Representatives of the US Army Engineer District, Los Angeles (SPL), observed testing of the final portion of Plan 1 and decided that a repeat test of this plan was not necessary.

15. Plan 2 (Photos 5-8 and Plate 5) was constructed in the same manner as Plan 1 except 13- to 20-ton stone was added to the sea side of the repair section. This stone was painted blue in the model so it could be differentiated from the stone used to represent the 1983 repair section (7- to 20-ton stone). The stone appears black in the photographs. The toe of the additional stone was placed 25 ft seaward from the toe of the 1983 repair. This distance is approximately 50 ft when measured horizontally from the center line of the breakwater. The 13- to 20-ton stone transitioned into the 7- to 20-ton stone while maintaining a maximum crest elevation of +13 ft mllw. It was hoped that flattening the slope and moving the toe into deeper water would increase the stability of the section. Stone placed along the toe on the 1V on 3H slope of the base mound appeared to be very vulnerable to wave attack. In fact, it was difficult to place the toe stone during construction as it tended to roll downslope. The test section was subjected to Hydrograph A (Table 1). Damage to the new stone section was initiated during Step 1 (6-sec, 8-ft waves, 0-ft mllw) and originated at the toe. Continued wave attack at the 0-ft mllw caused severe damage to the added stone in the rubble-mound section; however, the additional stone protected the stone representing the 1983 repair. Testing at the +8 ft swl caused extensive damage to the stone from the 1983 repair with damage initiating at Step 6 (6-sec, 14-ft

waves) and progressing with continued wave attack. Damage to the existing granite stone was also initiated during attack of 6-sec, 14-ft waves at the +8 ft swl. Wave attack during Step 8 (11-sec, 16-ft waves) caused significant damage to the granite stone, and extensive damage was done during Step 10 (22-sec, 16-ft waves) with this section failing completely. The additional stone added some reserve stability to the rubble-mound section and there was more stone left after testing than for Plan 1. Testing was stopped after Step 10 of Hydrograph A, because it was determined unnecessary to continue after complete failure of the existing granite block sections had taken place.

16. Plan 3 (Photos 9-14 and Plate 6) was similar to Plan 2 except a trench was excavated along the toe of the added 13- to 20-ton stone. The trench was about 7 ft wide and 3.5 ft deep, i.e., wide enough to allow a 20-ton stone to be placed inside at a depth of approximately one-half that of the stone. After the 13- to 20-ton stones were placed in the trench, voids between the stones were backfilled with material left from the excavation. A total of 478 7- to 20-ton stones and 246 13- to 20-ton stones were placed in the model. The test section was subjected to Hydrograph A. Wave attack at the 0-ft swl caused minor damage to the rubble-mound section with the first stones being displaced during attack of the 6-sec, 12-ft waves (Step 2). Most of the movement occurred in the transition areas. Wave attack at this swl caused no damage to the existing granite stone sections, and only one stone was displaced from the area representing the 1983 repair. During wave attack at the +8 ft swl, several additional stones were randomly displaced throughout the rubble-mound section, but the total damage to this area was still minor. Damage to the existing granite block stone was initiated during Step 6 (6-sec, 14-ft waves) and became extensive during Step 8 (11-sec, 16-ft waves). This area of the structure failed completely. Testing was stopped after completing Step 8 of Hydrograph A. The test section was rebuilt and subjected to Hydrograph B with similar results. Damage was initiated during Step 5 (8-sec, 16-ft waves, 0-ft mllw) for the 13- to 20-ton stone and during Step 8 (6-sec, 14-ft waves, +8 ft mllw) for the existing granite block sections. Damage to the existing granite block sections was significant during Step 9 (8-sec, 16-ft waves, +8 ft mllw) with these sections failing completely during tests at the +8 ft swl. Hydrograph B was run to completion with the rubble-mound section sustaining only minor damage except in the transition areas where stone was lost after the existing granite block sections had failed

completely. Based on these test results, it can be concluded that Plan 3 offers a repair option that is more stable than the original breakwater.

17. Plan 4 (Photos 15-18 and Plates 7-8) was the same as Plan 3 except the toe stone on the sea side of the repair section was buttressed with two layers of 3- to 4-ton angular stone rather than being placed in a trench. Placement of the 3- to 4-ton stone started at the toe of the base mound, -47 ft contour, and then continued upslope to the point where the 13- to 20-ton stone started. The 3- to 4-ton stone bordered the 13- to 20-ton stone in the transition areas (see Plate 8). A total of 475 7- to 20-ton stones, 245 13- to 20-ton stones, and 1,746 3- to 4-ton stones were placed in the model. The test section was subjected to Hydrograph B. Wave attack at the 0-ft swl caused minor damage to the 3- to 4-ton stone with most of the smaller stone in the transition areas being displaced. The 3- to 4-ton stone along the toe of the 13- to 20-ton stone remained stable. Wave attack at the 6 ft swl caused minor damage to the 13- to 20-ton stone with most damage occurring during attack of 8-sec, 16-ft waves; displacement was concentrated in the transition areas. Damage to the existing granite block sections was initiated during Step 9 (8-sec, 16-ft waves, +8 ft mllw) with these sections failing completely during Step 10 (11-sec, 16-ft waves, +8 ft mllw). Wave attack at the +8 ft swl caused minor damage to the rubble-mound section because of overtopping. Hydrograph B was run to completion with some additional damage occurring in the transition areas where stone was lost after the existing granite block sections had failed completely. Based on these test results, it can be concluded that Plan 4 offers a repair option that is more stable than the original breakwater but less stable than Plan 3.

Tests for a 45-deg angle of wave attack

18. Plan 3 (Photos 19-22 and Plate 6) was selected for testing at wave direction 2 because it appeared to be the best alternative based on constructability and economic considerations. As requested by representatives of SPL, the granite block sections were wired in place so the stability of the repair section could be checked independent of the original structure. The test section was subjected to Hydrograph B. Wave attack at the 0-ft mllw caused slight movement as the stone shifted and then nested after construction. Testing at the +6 ft mllw caused minor damage to 13- to 20-ton stone at the seaward (left) transition and slight damage to 13- to 20-ton stone at the

shoreward (right) transition. There was also minor damage, caused by overtopping, of the 7- to 20-ton stone on the harbor side of the structure. Wave attack at the +8 ft swl caused moderate damage to the harbor-side 7- to 20-ton stone. Most of the waves overtopped the structure at this swl and there was minimum wave energy concentrated in th^ sea-side transition areas. There was slight change in the damage level as the hydrograph was run to completion. Based on these test results, it can be concluded that Plan 3 offers a more stable repair option for both wave directions 1 and 2 than the original breakwater. There was minor damage to the 13- to 20-ton stone at the seaward transition during testing from wave direction 2, but the structural integrity of this area remained intact. It appears that if there was a greater duration of long-period waves than tested at the +8 ft mllw level the 7- to 20-ton stone on the harbor side of the structure might be lost; however, the original granite block sections would fail before this happens.

Safety Factor Tests

19. When designing breakwaters, as with any engineered structure, it is advantageous to determine the margin of safety for selected designs. Consequently, it was decided to investigate the stability response of Plans 3 and 4 for wave heights in excess of the 16-ft design height. A review of calibration data for the +8 ft swl revealed that the 8- and 11-sec wave periods became steepness-limited at heights of 20 and 24 ft, respectively. Tests wave heights were restricted also because the wave generator reached its stroke limit at 16-sec, 20-ft and 22-sec, 16-ft waves. Wave conditions used for the safety factor tests are listed in Table 2. Details of the plans tested are described in the following paragraphs.

20. Plan 3 (Plate 6) was tested to determine its margin of safety for both wave directions 1 and 2. Safety factor tests (Table 2) of Plan 3 were initiated as an extension of Hydrograph B (Table 1). For wave direction 1, damage progressed throughout the safety factor tests; however, failure was not complete at the conclusion of tests (Photos 23 and 24). Damage was concentrated in two areas, near the transitions and the harbor side of the repair section. Wave overtopping caused displacement of the 7- to 20-ton stone on the harbor side of the structure with the largest amount of dislocation occurring during attack of 8-sec, 20-ft waves. Original granite blocks had

been displaced during Hydrograph B, leaving the stone in the transition areas without support and vulnerable to wave attack during safety factor tests. Damage from testing at wave direction 2 was concentrated on the harbor side of the repair section and was caused by wave overtopping. The 7- to 20-ton stone on the harbor side was displaced first, leaving the 13- to 20-ton stone on the sea side without support and vulnerable to wave attack. Damage became extensive for the 16-sec, 18-ft waves. Although an attempt was made to hold the original granite block sections in place for testing at wave direction 2, some granite block was displaced in the transition areas. Again, failure of the repair section was not complete at the conclusion of tests (Photos 25 and 26).

21. Results of safety factor tests show Plan 3 has sufficient resiliency to withstand waves in excess of design height without experiencing complete failure.

22. Plan 4 (Plates 7 and 8) was tested to determine its margin of safety from wave direction 1. Safety factor tests were initiated as an extension of Hydrograph B. Results of the tests were similar to those for Plan 3 from wave direction 1. Damage was concentrated in two areas, near the transitions and the harbor side of the repair with most of the damage caused by overtopping. Original granite blocks had been displaced during Hydrograph B, leaving the stone in the transition areas without support and vulnerable to wave attack during the safety factor tests.

23. Results of safety factor tests (Photos 27 and 28) show Plan 4 has sufficient resiliency to withstand waves in excess of design height without experiencing complete failure and confirms the conclusion from previous tests that Plan 4 exhibits a response similar to but slightly less than Plan 3 for wave direction 1.

PART IV: CONCLUSIONS

24. Based on the test results, and observations presented herein, it is concluded that:

a. For the selected design conditions of 6- to 22-sec, 8- to 16-ft waves at swls of 0 to +8 ft mllw and a wave direction of 90 deg:

   (1) Plan 1 was unacceptable because damage of the repair section was initiated at lower wave heights than for the original breakwater; however, it should be noted that damage of angular stone progressed at a slower rate than that for the existing granite blocks and the angular stone exhibited more resiliency.

   (2) Plan 2 was not acceptable.

   (3) Plans 3 and 4 were acceptable for 90-deg wave attack, but Plan 3 was selected as the best alternative based on constructability and economic considerations.

b. For the selected design conditions, Plan 3 was acceptable for 45-deg wave attack.

c. For Plan 3, which was subjected to both 90- and 45-deg wave attack, the 90-deg wave direction proved to be the more severe condition for stability of the sea-side stone while the 45-deg wave direction was more severe for the angular stone located in the transition areas of the repair section.

d. Safety factor tests showed that both Plans 3 and 4 provided a more stable structure than the original breakwater.

Table 1

Hydrographs A and B

Step	Still-Water Level ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr
		Hydrograph A		
1	0	6	8	0.25
2	0	6	10	0.25
3	0	6	12	0.25
4	0	6	14	0.50
5	0	8	16	1.00
6	+8	6	14	1.00
7	+8	8	16	2.00
8	+8	11	16	0.50
9	+8	16	16	0.50
10	+8	22	16	0.50
11	0	8	16	1.00
12	0	6	14	1.00
		Hydrograph B		
1	0	6	8	0.25
2	0	6	10	0.25
3	0	6	12	0.25
4	0	6	14	0.50
5	0	8	16	1.00
6	+6	6	14	0.50
7	+6	8	16	1.00
8	+8	6	14	1.00
9	+8	8	16	2.00
10	+8	11	16	0.50
11	+8	16	16	0.50
12	+8	22	16	0.50
13	+6	8	16	1.00
14	+6	6	14	1.00
15	0	8	16	1.00
16	0	6	14	1.00

Table 2

Safety Factor Test

Step	Still-Water Level ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr
1	+8	8	18	0.50
2		11	18	
3		16	18	
4		8	20	
5		11	20	
6		16	20	
7		11	22	
8		11	24	

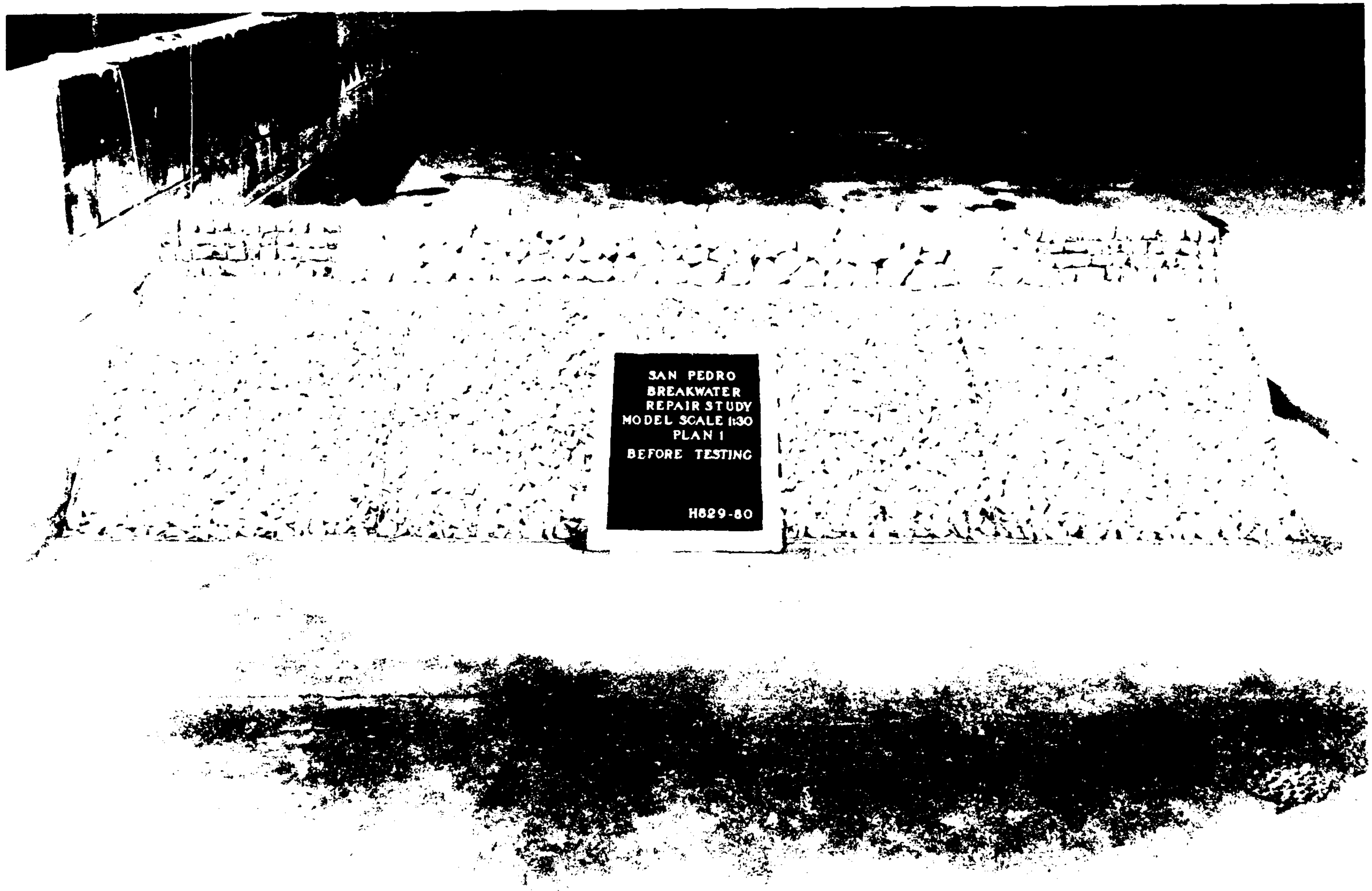


Photo 1. Sea-side view of Plan 1 before wave attack

Photo 2. Harbor-side view of Plan 1 before wave attack

Photo 3. Sea-side view of Plan 1 after testing with Hydrograph B.
Angle of wave attack = 90 deg

Photo 4. Harbor-side view of Plan 1 after testing with Hydrograph B.
Angle of wave attack = 90 deg

Photo 5. Sea-side view of Plan 2 before wave attack

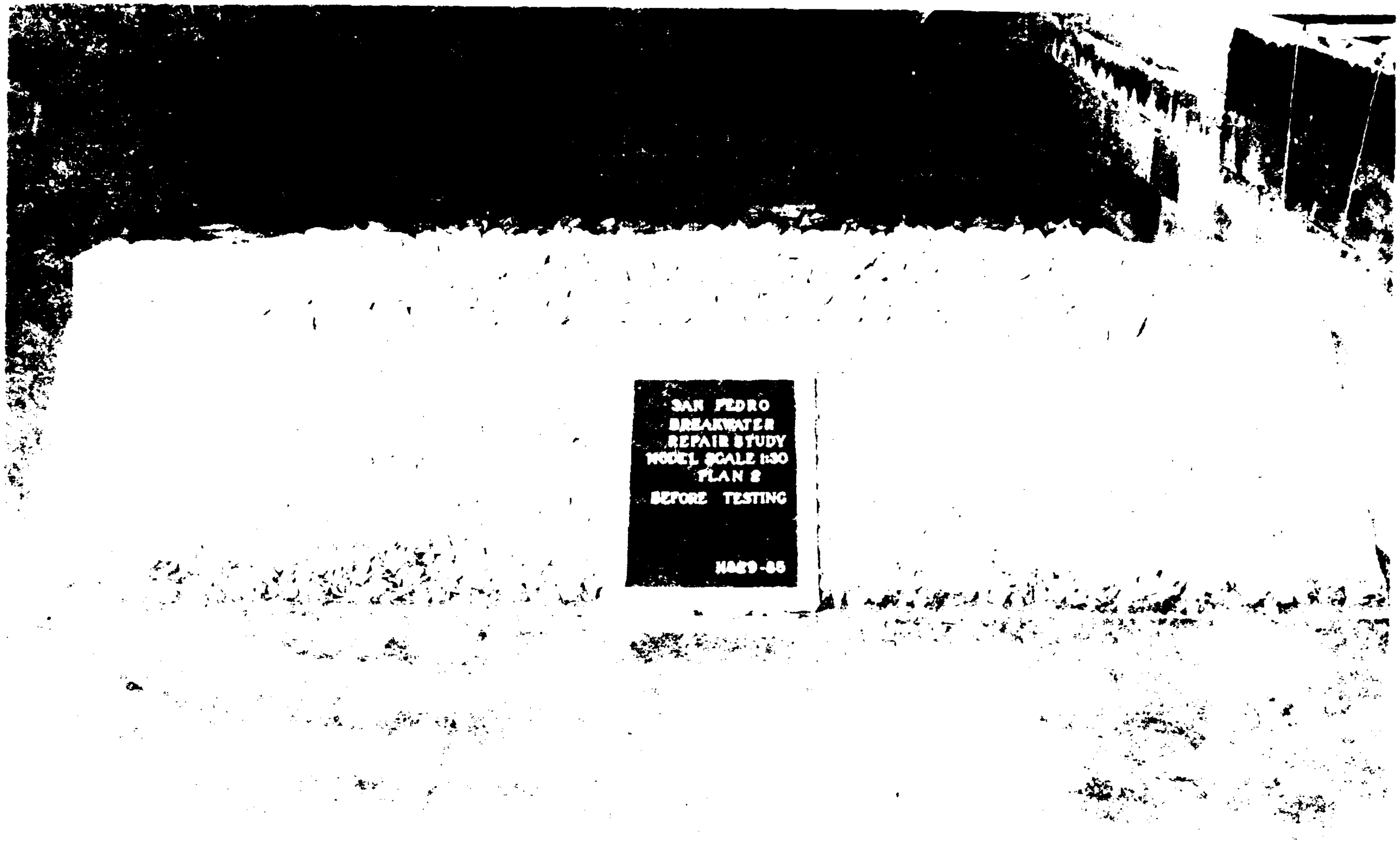


Photo 6. Harbor-side view of Plan 2 before wave attack

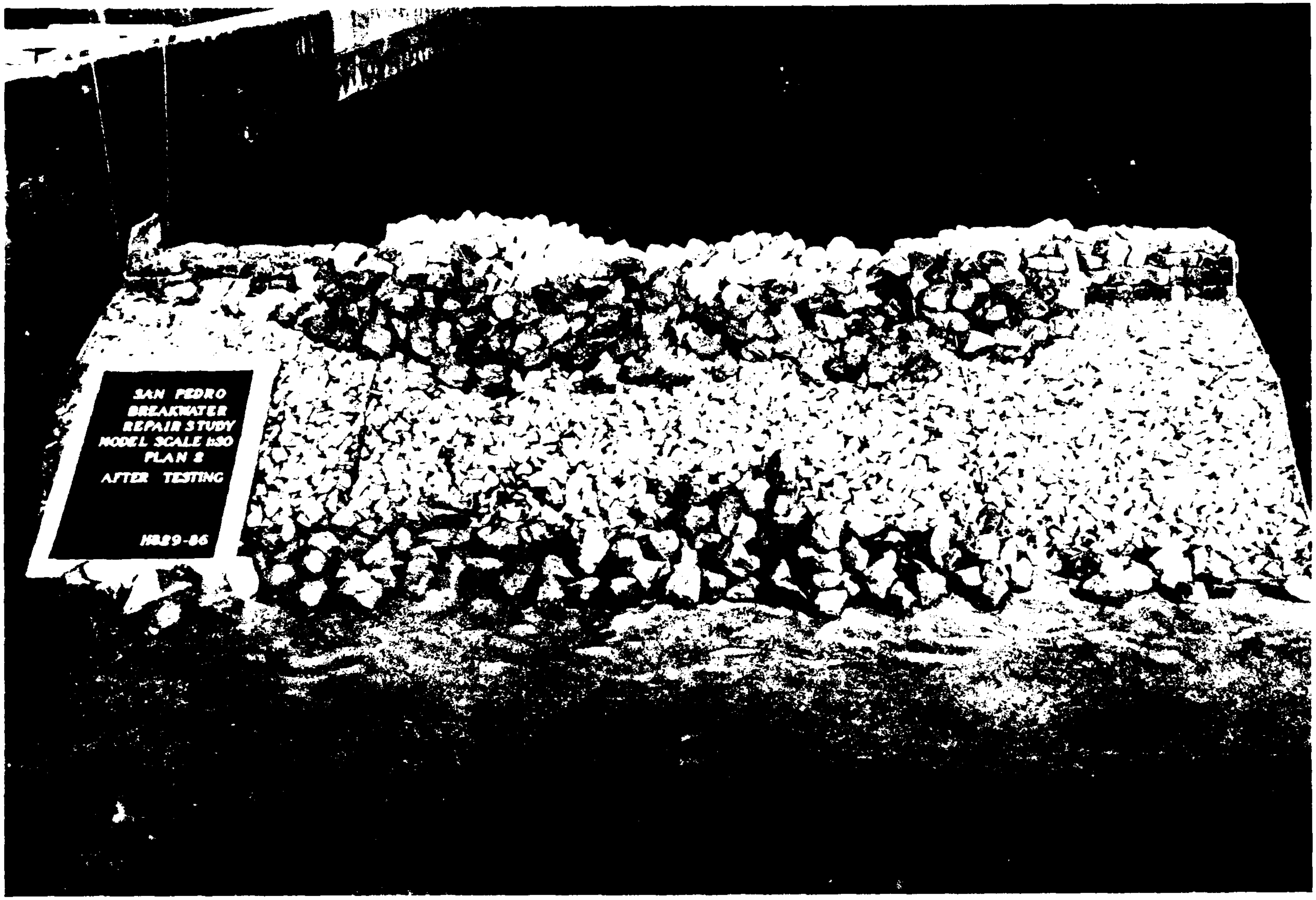


Photo 7. Sea-side view of Plan 2 after subjection to Hydrograph A. Angle of wave attack = 90 deg

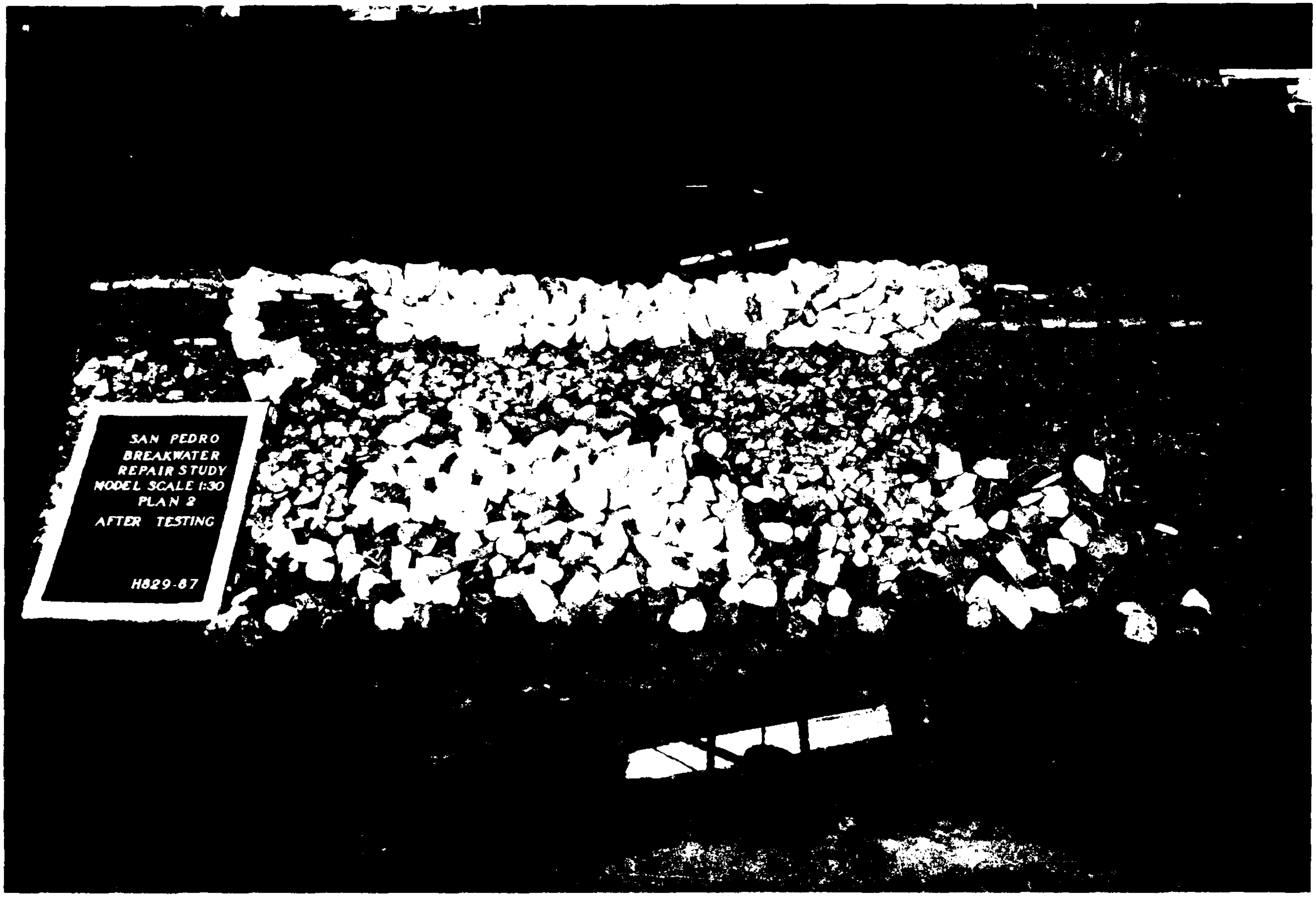


Photo 8. Harbor-side view of Plan 2 after subjection to Hydrograph A. Angle of wave attack = 90 deg

Photo 9. Sea-side view of Plan 3 before wave attack

Photo 10. Harbor-side view of Plan 3 before wave attack

Photo 11. Sea-side view of Plan 3 after subjection to Hydrograph A. Angle of wave attack = 90 deg

Photo 12. Harbor-side view of Plan 3 after subjection to Hydrograph A. Angle of wave attack = 90 deg

Photo 13. Sea-side view of Plan 3 after subjection to Hydrograph B. Angle of wave attack = 90 deg

Photo 14. Harbor-side view of Plan 3 after subjection to Hydrograph B.
Angle of wave attack = 90 deg

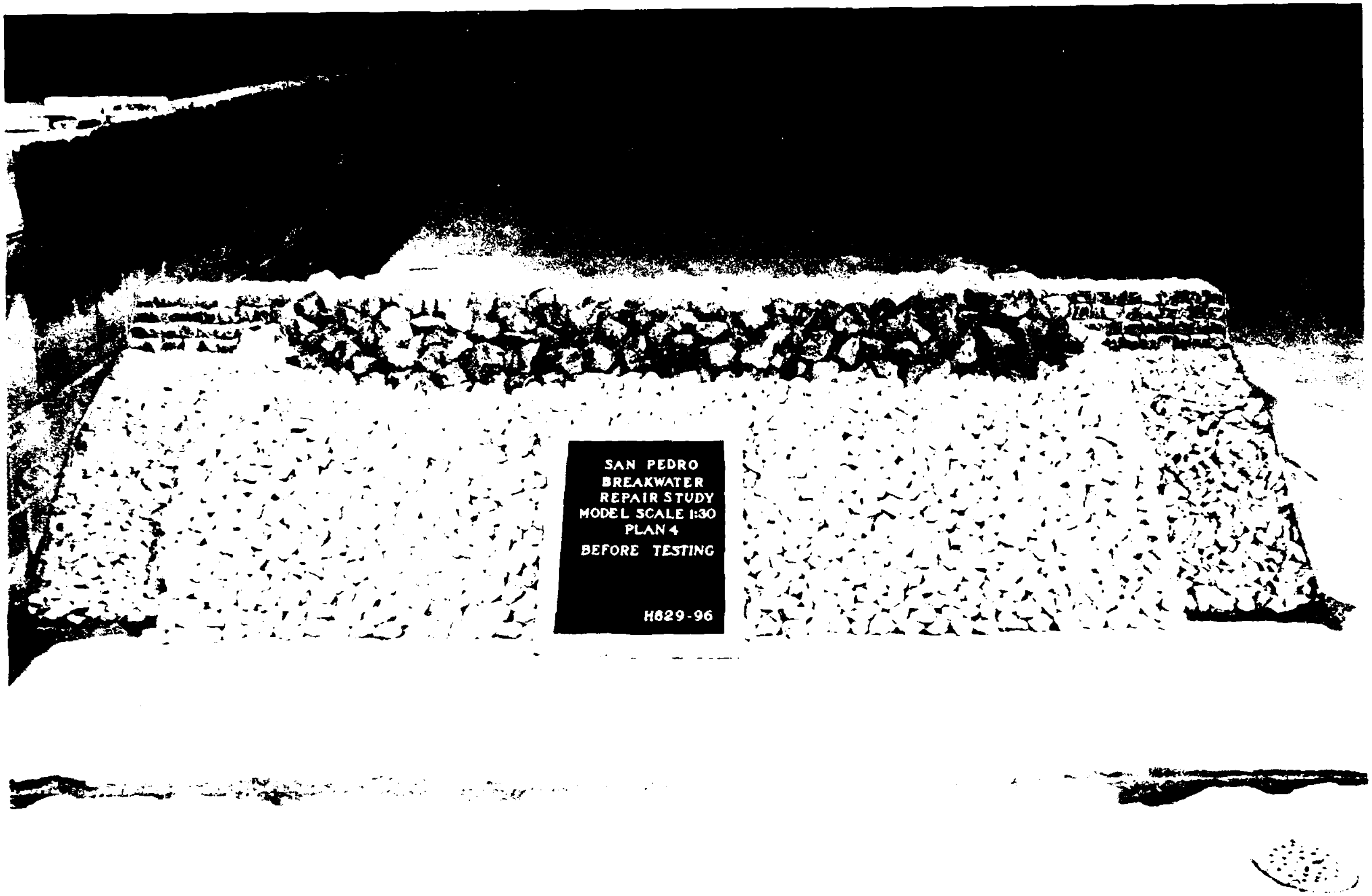


Photo 15. Sea-side view of Plan 4 before wave attack

Photo 16. Harbor-side view of Plan 4 before wave attack

Photo 17. Sea-side view of Plan 4 after subjection to Hydrograph B. Angle of wave attack = 90 deg

Photo 18. Harbor-side view of Plan 4 after subjection to Hydrograph B. Angle of wave attack = 90 deg

Photo 19. Sea-side view of Plan 3 before wave attack.
Angle of wave attack = 45 deg

Photo 20. Harbor-side view of Plan 3 before wave attack.
Angle of wave attack = 45 deg

Photo 21. Sea-side view of Plan 3 after subjection to Hydrograph B.
Angle of wave attack = 45 deg

Photo 22. Harbor-side view of Plan 3 after subjection to Hydrograph B.
Angle of wave attack = 45 deg

Photo 23. Sea-side view of Plan 3 after factor of safety tests.
Angle of wave attack = 90 deg

Photo 24. Harbor-side view of Plan 3 after factor of safety tests.
Angle of wave attack = 90 deg

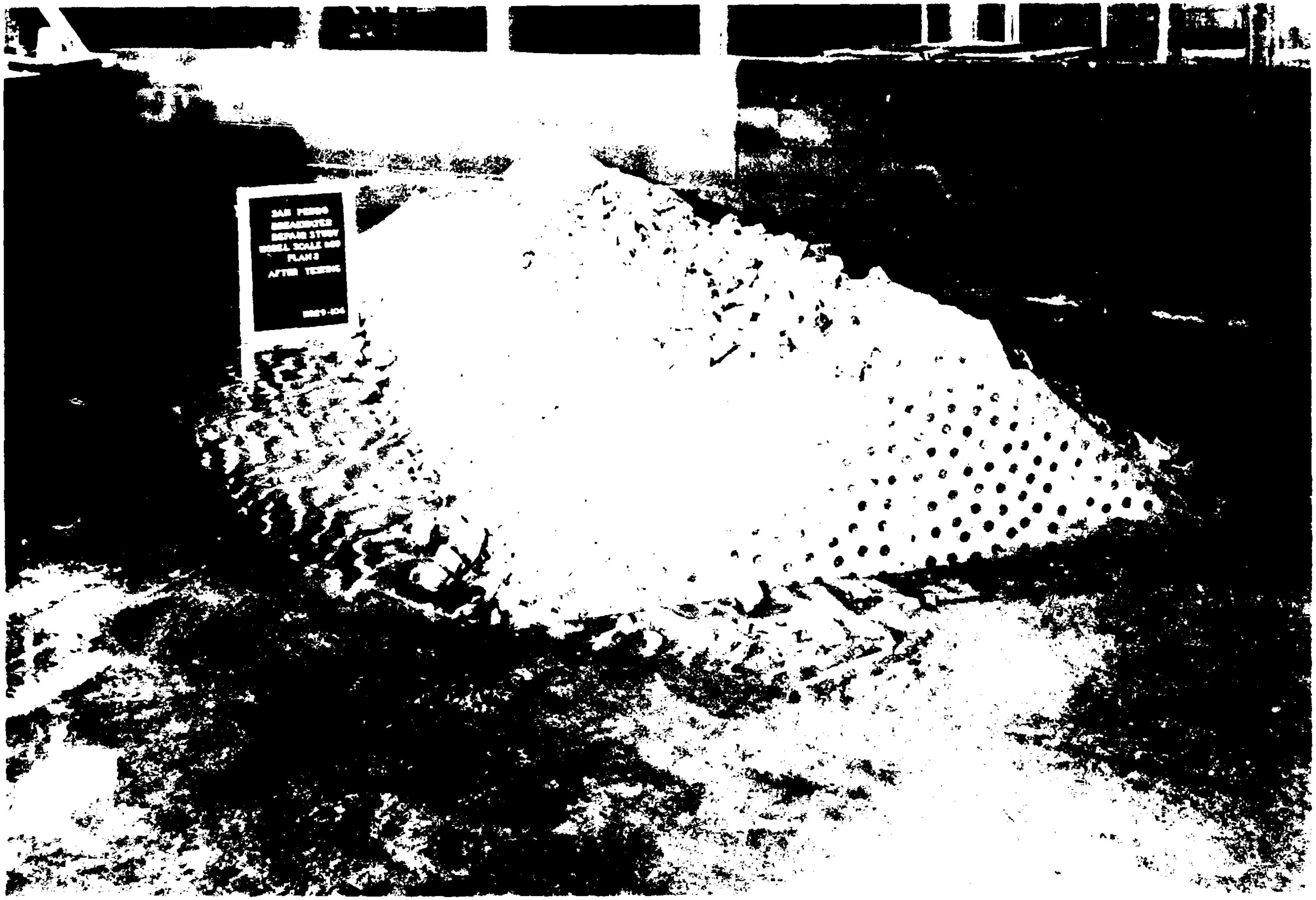


Photo 25. Sea-side view of Plan 3 after factor of safety tests.
Angle of wave attack = 45 deg

Photo 26. Harbor-side view of Plan 3 after factor of safety tests.
Angle of wave attack = 45 deg

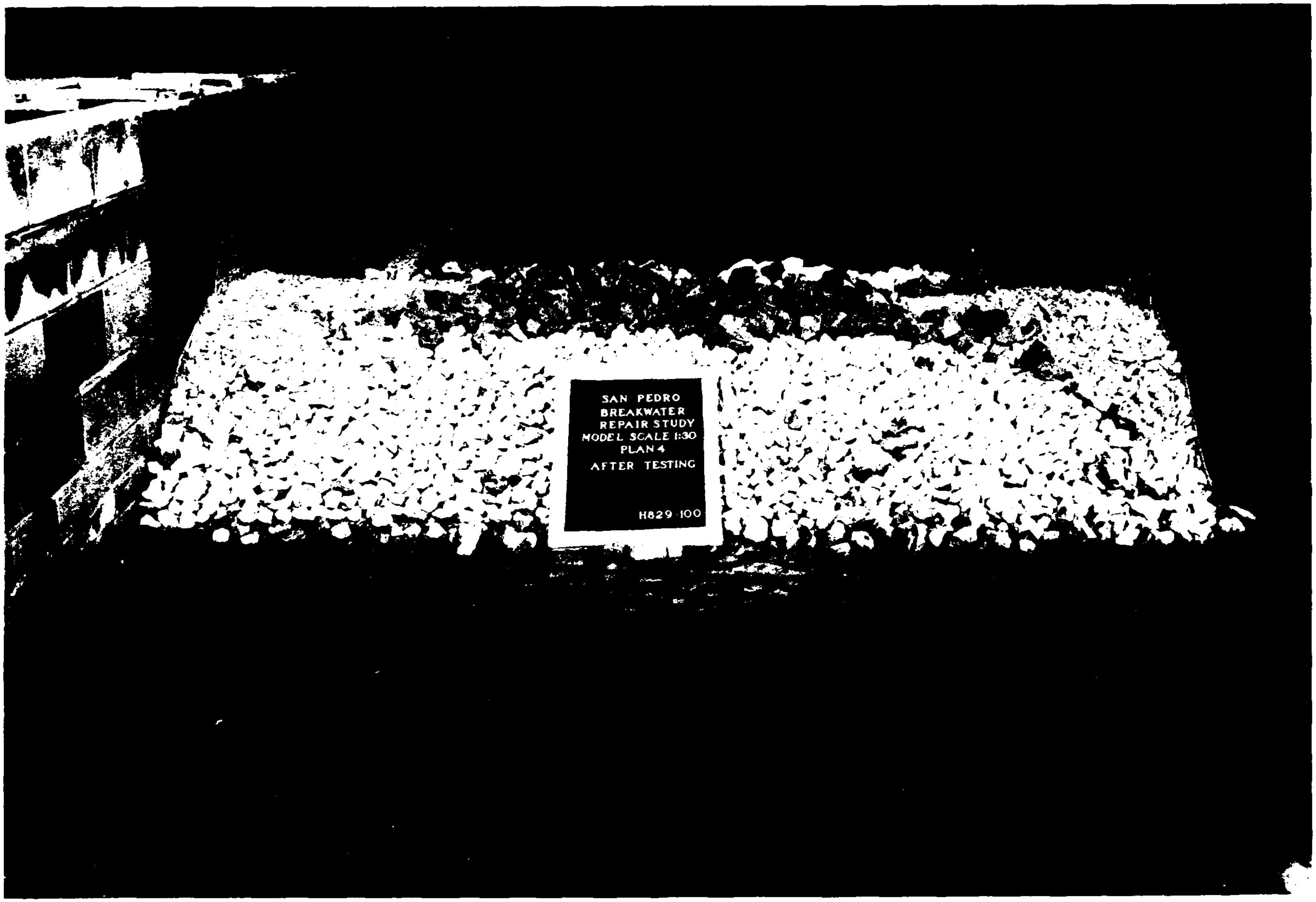
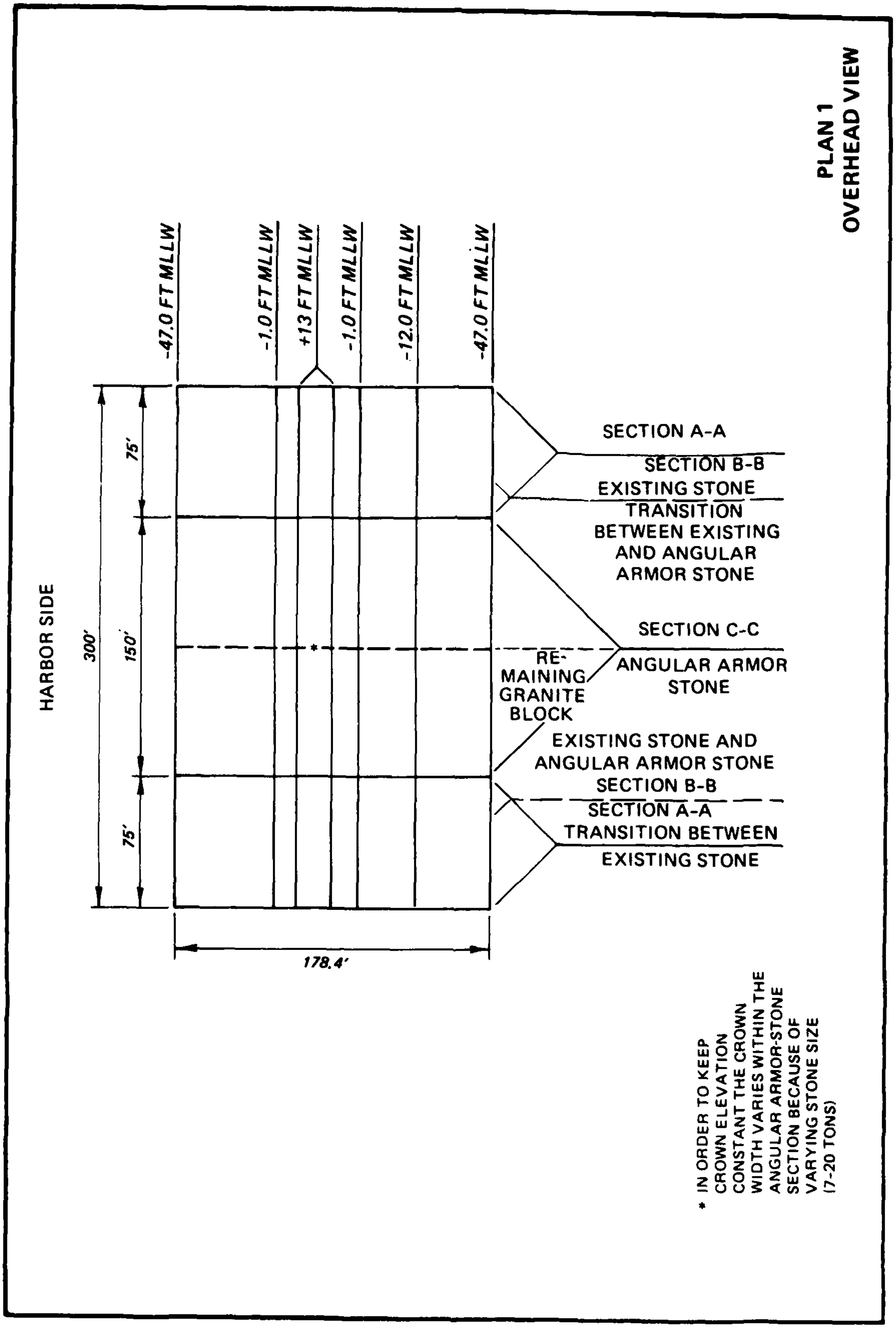
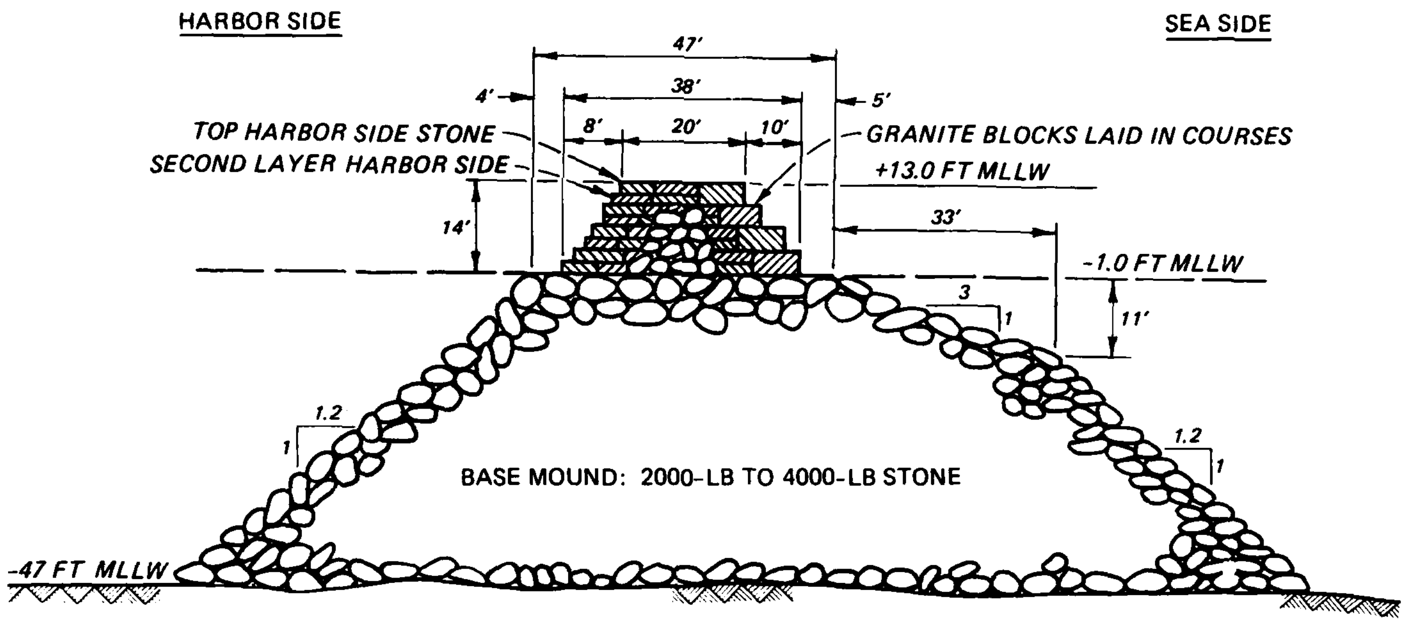


Photo 27. Sea-side view of Plan 4 after factor of safety tests.
Angle of wave attack = 90 deg

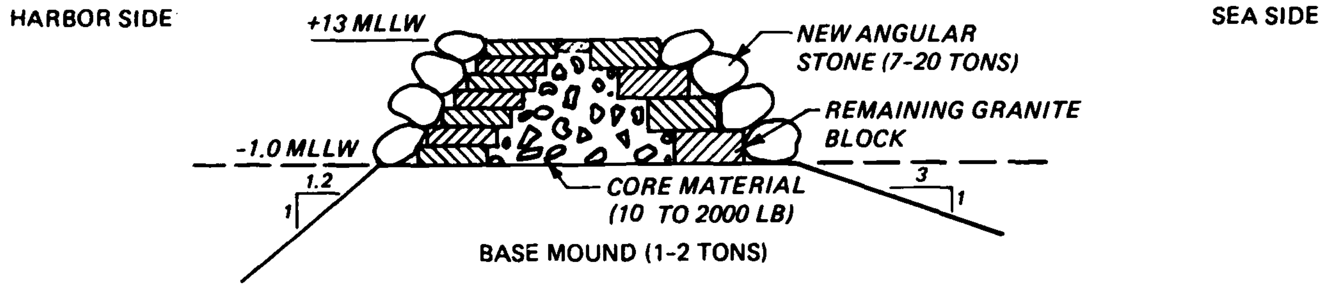
Photo 28. Harbor-side view of Plan 4 after factor of safety tests.
Angle of wave attack = 90 deg


PLAN 1
OVERHEAD VIEW
HARBOR SIDE
-47.0 FT MLLW
-1.0 FT MLLW
+13 FT MLLW
-1.0 FT MLLW
-12.0 FT MLLW
-47.0 FT MLLW
300'
75'
150'
75'
178.4'
SECTION A-A
SECTION B-B
EXISTING STONE
TRANSITION
BETWEEN EXISTING
AND ANGULAR
ARMOR STONE
SECTION C-C
ANGULAR ARMOR
STONE
RE-
MAINING
GRANITE
BLOCK
EXISTING STONE AND
ANGULAR ARMOR STONE
SECTION B-B
SECTION A-A
TRANSITION BETWEEN
EXISTING STONE
* IN ORDER TO KEEP CROWN ELEVATION CONSTANT THE CROWN WIDTH VARIES WITHIN THE ANGULAR ARMOR-STONE SECTION BECAUSE OF VARYING STONE SIZE (7-20 TONS)

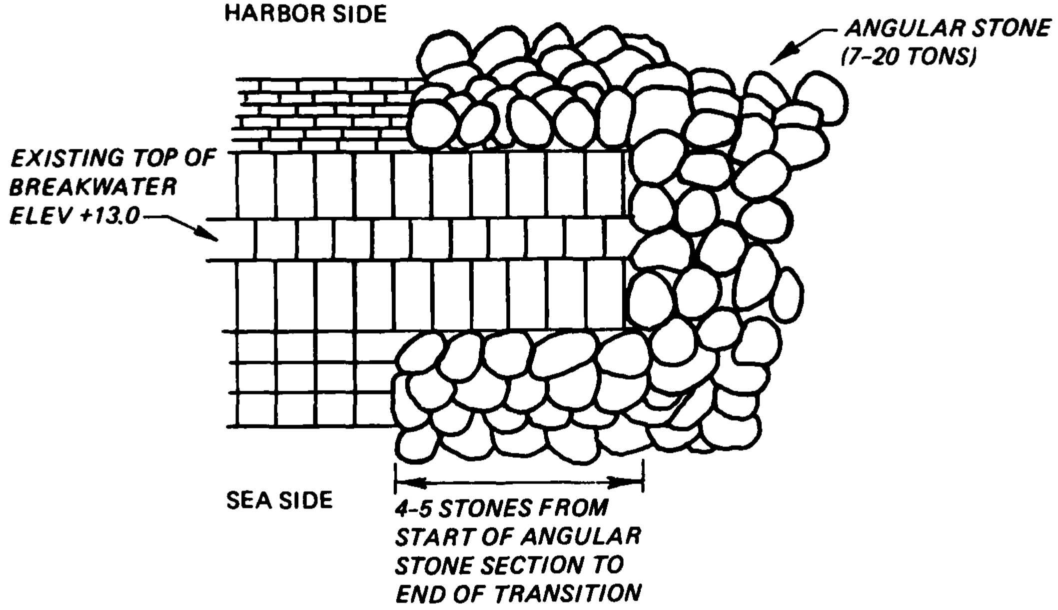


NOTE: 1. SEA-SIDE ARMOR STONE RANGES FROM 10 TO 12 TONS
2. HARBOR-SIDE ARMOR STONE RANGES FROM 5 TO 7 TONS
3. CORE MATERIAL PLACED ABOVE -1.0 FT MLLW RANGING FROM 10 TO 2,000 LB
4. UNIT WEIGHT OF STONE IS ASSUMED TO BE PCF

PLAN 1
SECTION A-A
EXISTING STONE CONSTRUCTION

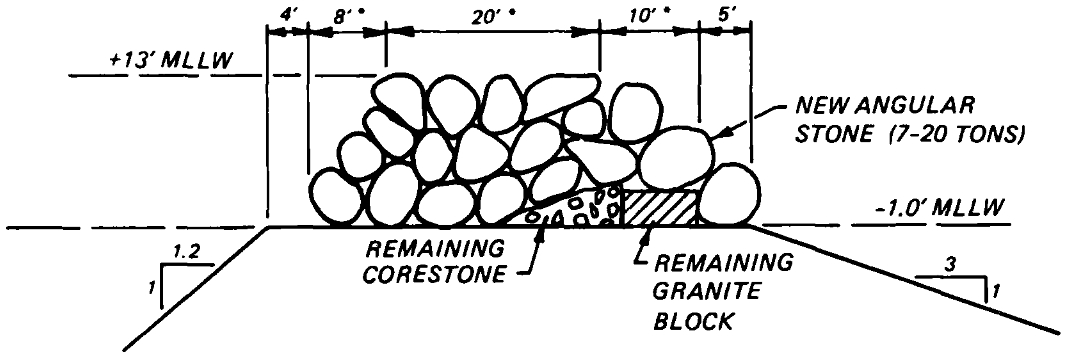


PROFILE OF TRANSITION AREA

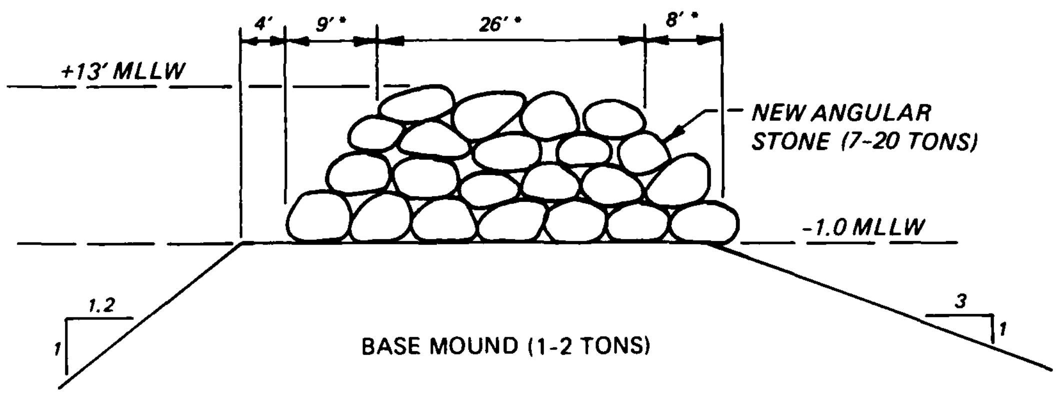


PLAN VIEW OF TRANSITION AREA

PLAN 1
SECTION B-B'
EXISTING AND ANGULAR ARMOR STONE



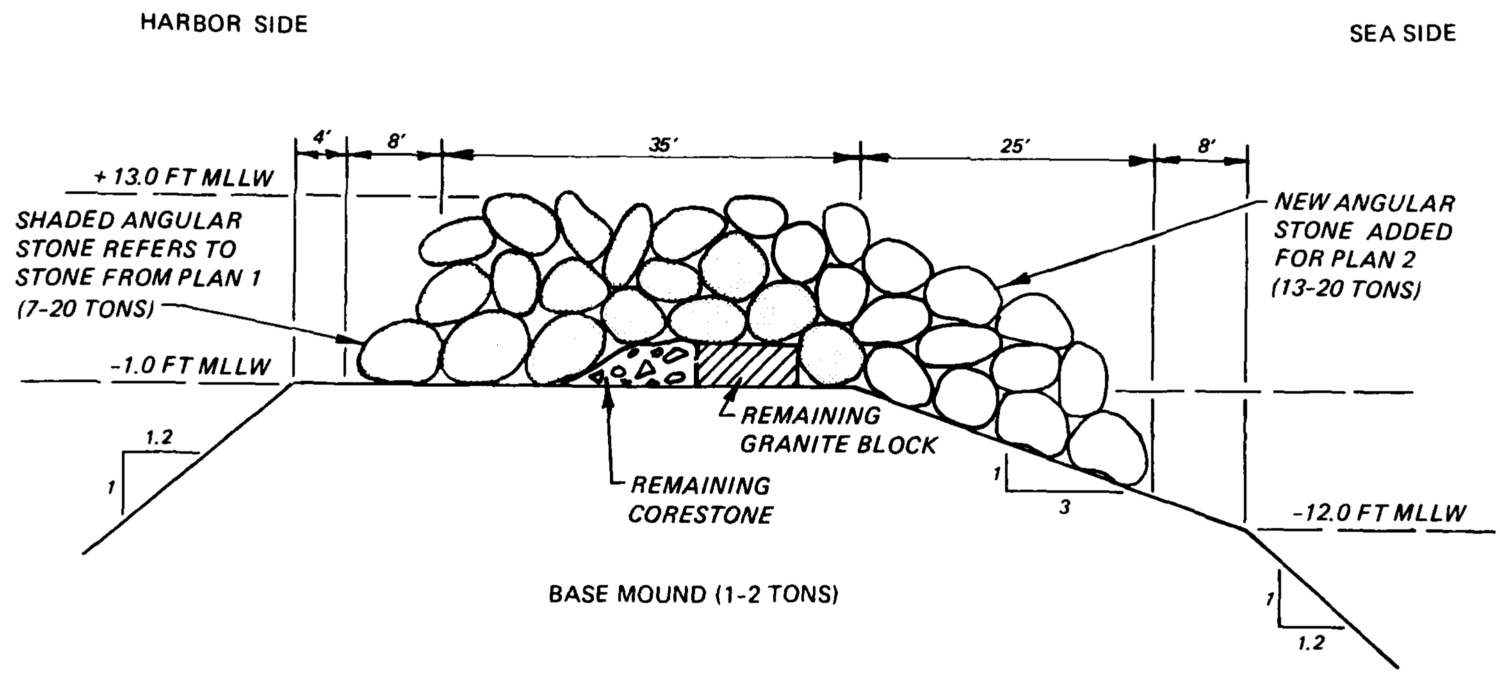
SECTION INCLUDING EXISTING STONE



SECTION WITHOUT EXISTING STONE

* IN ORDER TO KEEP CROWN ELEVATION CONSTANT THE CROWN WIDTH WILL VARY BE-CAUSE OF VARYING ARMOR STONE SIZE.

PLAN 1
SECTION C-C'
ANGULAR ARMOR STONE



PLAN 2
SECTION C-C'
ANGULAR ARMOR STONE

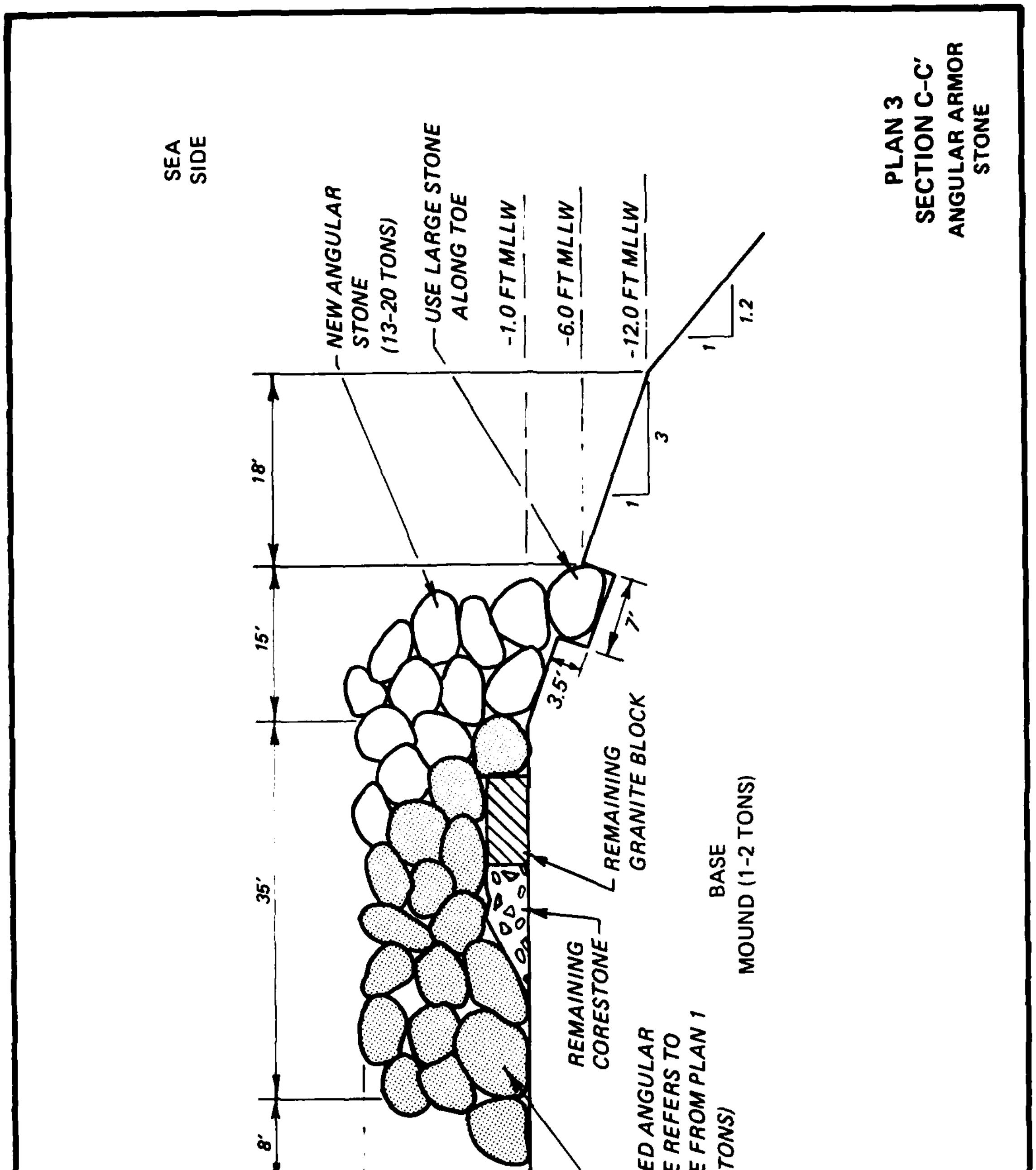
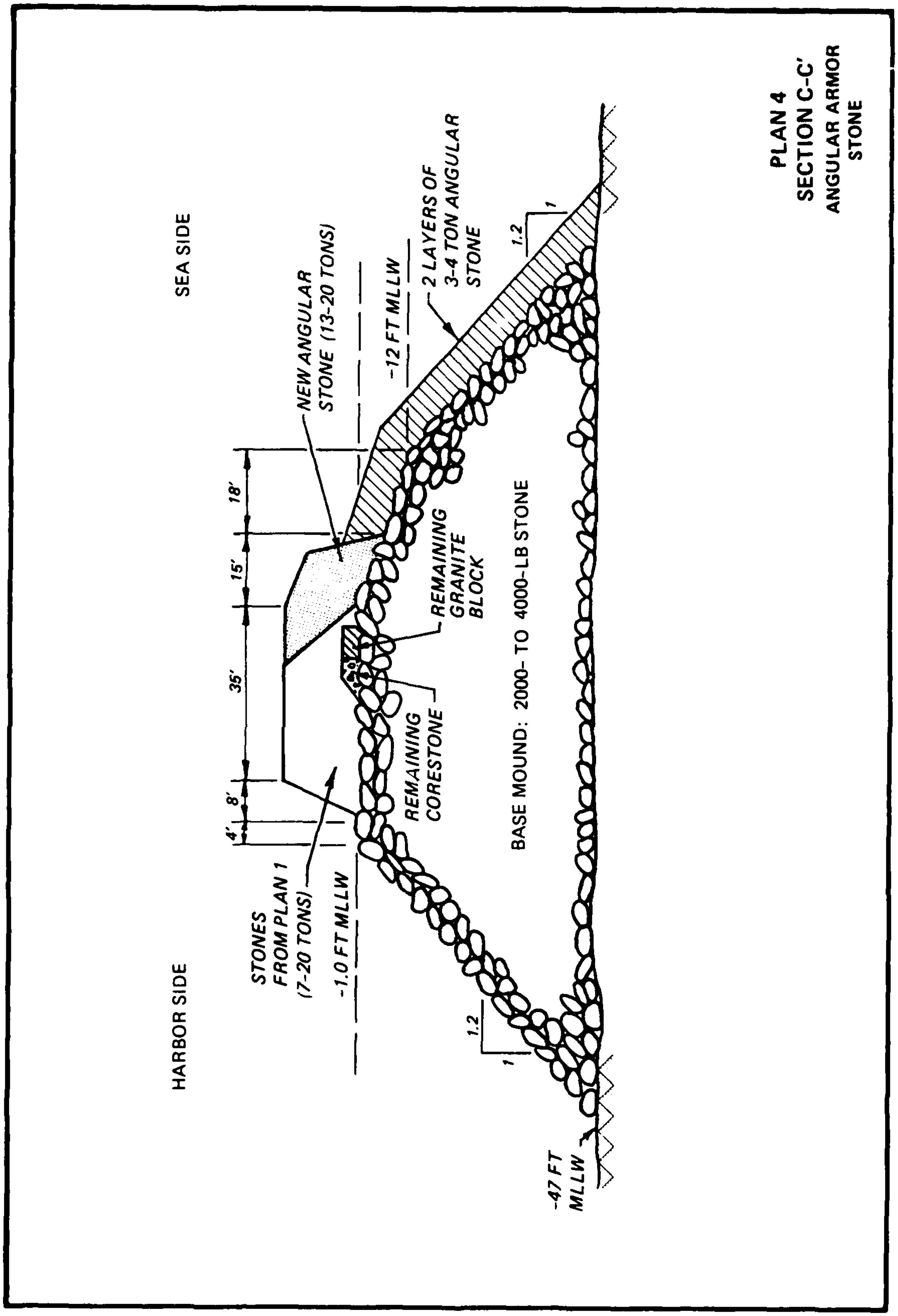
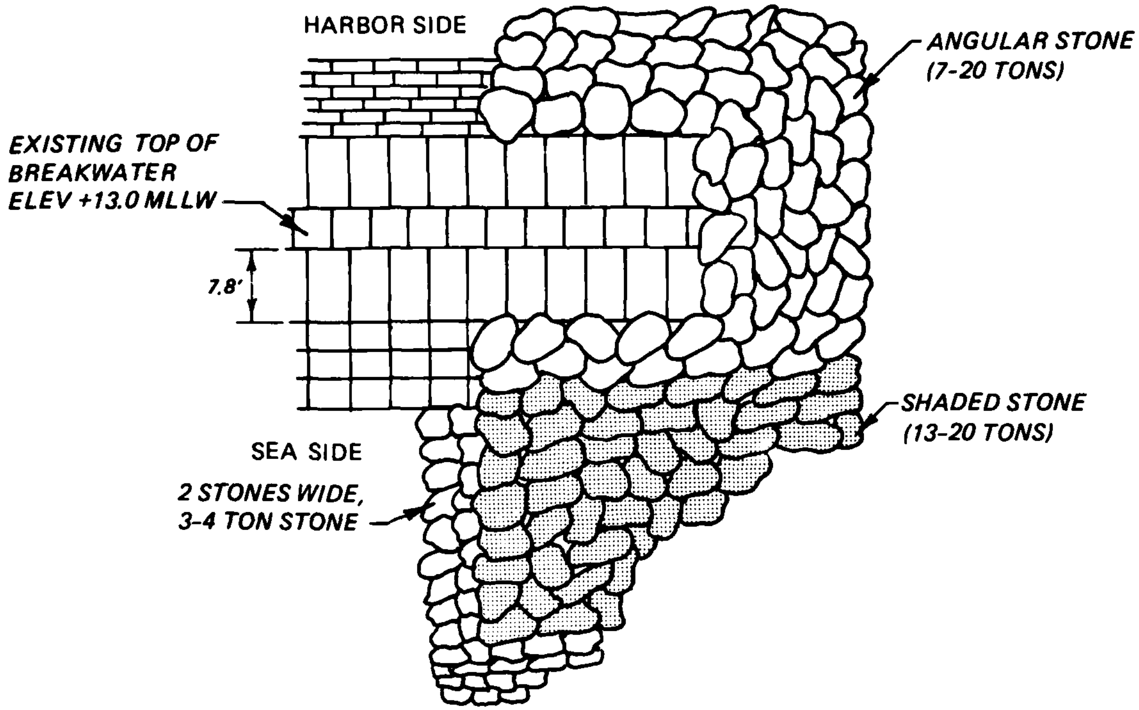

PLAN 3
SECTION C-C'
ANGULAR ARMOR STONE
SEA SIDE
HARBOR SIDE
NEW ANGULAR STONE (13-20 TONS)
USE LARGE STONE ALONG TOE
-1.0 FT MLLW
-6.0 FT MLLW
-12.0 FT MLLW
13.0 FT MLLW
18'
15'
35'
8'
4'
7'
3.5'
1
3
1.2
REMAINING GRANITE BLOCK
REMAINING CORESTONE
BASE MOUND (1-2 TONS)
SHADED ANGULAR STONE REFERS TO STONE FROM PLAN 1 (7-20 TONS)

PLATE 6


PLAN 4
SECTION C-C'
ANGULAR ARMOR STONE
SEA SIDE
HARBOR SIDE
NEW ANGULAR STONE (13-20 TONS)
-12 FT MLLW
2 LAYERS OF 3-4 TON ANGULAR STONE
1.2
1
18'
15'
35'
8'
4'
REMAINING GRANITE BLOCK
REMAINING CORESTONE
BASE MOUND: 2000- TO 4000-LB STONE
STONES FROM PLAN 1 (7-20 TONS)
-1.0 FT MLLW
1.2
1
-47 FT MLLW



PLAN VIEW OF TRANSITION AREA

PLAN 4
SECTION B-B'

TRANSITION BETWEEN EXISTING AND ANGULAR ARMOR STONE

END

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